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# Behavior and design of shingle joints, Oct. 1971

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# BEHAVIOR AND DESIGN OF SHINGLE JOINTS[a]

by

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# INTRODUCTION

Shingle joints have been used extensively in heavy built-up tension members to reduce the amount of splice material, and to facilitate the connection of the various components in large truss-type structures. These joints are designed using various approximations to determine the distribution of plate forces and the shear transfer along the joint length. The fasteners are then proportioned in single shear along the assumed critical shear planes. When high strength bolts were considered as a replacement for rivets in buildings, bridges and other steel structures, friction-type bolted joints were often used. These joints were considered comparable to riveted joints and did not take full advantage of the high shear strength of the bolts.

In this report both analytical studies and a complimentary experimental program covering a wide range of parameters are reported. The behavior of shingle-type joints in the working load range and in the non-linear range was examined, and existing methods for determining the approximate distribution of load were compared with the experimental load partitions. The object of the study was the development of design criteria that would provide the basis for more economical and safe design.

Fisher and Yoshida summarized the previous experimental and theoretical work on large riveted bridge joints.[6] They reported on the testing of two large shingle joints which simulated part of a chord member and splice from the Baton Rouge Interstate Bridge. One large joint was fastened with A325 bolts and the other with A502 Gr. 1 rivets. The work was limited to an evaluation of joint behavior in the elastic range, up to and including major joint slip. The testing was terminated when the machine capacity was reached.

Since it was not possible to determine the ultimate strength of the two large simulated bridge joints, the net cross sectional areas of the joints were reduced so that failure could occur within the machine capacity. The results of the re-testing of these modified joints were reported by Rivera and Fisher.[9] At ultimate load of the Modified Bolted Joint, there was considerable variation in the load carried by the individual fasteners. The study showed that the end fasteners in the first region were critical and that fasteners installed in the interior regions were not very effective. The Modified Riveted Joint appeared to provide a better redistribution of force to the interior regions due to the greater flexibility of the rivets. The bolted joint, however, was 27% stronger than the riveted joint. The end fasteners of both joints failed by unbuttoning.

Desai and Fisher reported on the development of a mathematical model for shingle joints that permitted the complete force-displacement relationship to be predicted up to the ultimate load.[3] Using the analysis, the ultimate strength of the Modified Bolted Joint and the Modified Riveted Joint were predicted within an accuracy of 3.9% and 8.5% respectively.

The previous experimental studies on the Simulated Bridge Joints[6] and the Modified Joints[9] were of exploratory nature. Further studies were required to evaluate in detail the ultimate strength characteristics of shingle joints so that the full range of behavior would be known and to provide further experimental confirmation for the theoretical solution for shingle joints.[3]

## PRELIMINARY ANALYTICAL STUDIES

The theoretical analysis for shingle joints developed in Ref. 3 was used to study analytically the effects of various joint geometries on the ultimate strength. The non-dimensionalized ratio of the predicted ultimate strength to the working load of the joint, $P_u/P_w$, was used as an index of joint behavior.

The idealized joints were assumed to have A572 steel plates fastened by A325 high strength bolts. The yield stress and ultimate tensile strength of the plates were taken to be 60 ksi and 88 ksi, respectively. The working loads for the joints were determined from the main plate net areas.

The variables studied were (a) the ratio $A_n/A_s$, defined as the ratio of the net main plate area in the first region to the total effective fastener shear area; (b) the total number of fasteners, N; (c) the number of fasteners per region; and (d) the number of regions.

Joint Behavior:- Figure 1 shows the change in joint strength with length for shingle joints with three equal regions for values of $A_n/A_s$ ranging from 0.375 to 1.00. This corresponds to a variety of allowable shear stresses. Ratios of $A_n/A_s$ between 0.375 and 0.5 are typical of current friction-type joints. An $A_n/A_s$ ratio of 0.625 corresponds to a shear stress of 22 ksi. In the analysis, the fasteners were assumed to act in double shear in all three regions. As observed in previous studies of butt joints, a decrease in average joint strength occurred with an increase in length.[4,5]

Figure 1 indicates that only minor changes in joint strength beyond the working load resulted in spite of substantial variations in joint proportions. A 40% increase in plate capacity between the 0.625 and 0.875 $A_n/A_s$ ratio only resulted in a 10% decrease in the $P_u/P_w$ ratio. This indicates that the same number of fasteners are capable of satisfactory behavior at allowable shear stresses substantially higher than used in current practice for bearing-type joints.

The strengths of the joints summarized in Fig. 1 were predicted assuming double shear behavior throughout the joint length. In Fig. 2, the effect of assuming only single shear in the interior regions and its effect upon the predicted ultimate strengths is shown.

At the lower $A_n/A_s$ ratios, the predicted strengths of the joints were comparable for both idealizations of joint behavior. At higher $A_n/A_s$ levels the load carried by interior fasteners was greater and an assumed reduction in effective shear area had a more pronounded influence on joint strength.

Figure 3 compares the computed fastener stress assuming double shear in a shingle joint with an $A_n/A_s$ ratio of 0.50 with the computed stress assuming single shear in the interior regions. The predicted ultimate strength was reduced only slightly since comparable behavior occurred in the first region which was critical. The amount of load transferred in each region was about the same, however, the stress in the interior regions was nearly doubled when only one shear plane was assumed to be effective. Corresponding to the reduction in effective shear area was an increase in $A_n/A_s$ ratio from 0.50 to 0.75.

<u>Variation in Region Length</u>:- A study was made to determine the effects of varying the number of fasteners in each region. The total number of fasteners and the plate areas were maintained, but the region lengths were adjusted by shifting an equal number of fasteners from each interior region into the first region. Double shear was assumed in Region 1 with single shear in the interior regions. In certain cases, fastener failure was predicted in the interior regions rather than at the end of the joint when the fasteners were rearranged.

Overall it was found that the predicted strength of shingle joints of a given length was not greatly influenced by minor variations in the number of fasteners in each region.

Number of Regions:- A study was made to determine the effect of varying the number of main plate terminations, i.e., the number of regions in joints. Joints with one, two and three regions and the same number of fasteners were compared. Double shear was assumed in the first region with single shear in the interior regions. The one-region joints were symmetrical butt joints having the total main plate area terminated at one location. In the two-region joints, the main plate area was terminated in equal amounts at two separate locations. The three-region joints had the geometry shown in Fig. 2.

At the 0.5 $A_n/A_s$ ratio, little variation in strength was predicted by changing the number of regions. At higher $A_n/A_s$ ratios, the distribution of load in the interior fasteners is greater than at lower $A_n/A_s$ ratios. Thus, terminating the main plates at different locations and thereby reducing the effective shear area causes a reduction in predicted ultimate strength.

## DESCRIPTION OF TESTS

A test program consisting of nine shingle joints was developed on the basis of the preliminary analytical studies. The program was intended to show experimentally that the ultimate strength trends predicted by the analytical studies were valid, and to further verify the theoretical solution for the strength of shingle joints.[3]

The geometry of the test joints is summarized in Table 1. For each joint, the total number of fasteners, the number of fasteners in each region, the overall lengths, the individual region lengths and the gage distances are given. Joints 1, 2, 5, 7 and 8 were 3-region joints intended to show experimentally the effects of removing fasteners from the interior regions, and shifting fasteners from the interior regions into the first region. Joints 7 and 8 were comparable to a gage strip from the Modified Bolted Joint reported in Ref. 9. Joint 7 had twice as many fasteners as the Modified Bolted Joint, and Joint 8 had twice the number in the first region. The joint areas were comparable. Joints 3 and 4 were designed as 2-region joints intended to show the effect of the number of regions and variations in $A_n/A_s$ ratio. Joint 3B was identical to 3A except for the type of fastener. Huck fasteners were used in Joints 2 and 3B. All other joints were fastened with 7/8 in. A325 bolts.

The fabrication procedure for the test joints was the same as in previous work. Bolting-up operations were carried out at Fritz Engineering Laboratory, Lehigh University. The turn-of-nut method of tightening was used for the joints fastened with A325 bolts. Bolt elongations were measured after tightening to determine the clamping forces.[10] The Huckbolts were installed with equipment furnished by the Huck Manufacturing Company. A detailed description of the Huckbolt is given in Ref. 7.

The plates used for the experimental program were A572 Grade 50 structural steel from the same heat. The mean static yield was 49.0 ksi with a standard deviation of 2.3 ksi and the mean tensile strength was 79.0 ksi with a standard deviation of 3.1 ksi.

Three separate lots of A325 bolts were used. Bolts with a 5 in. grip were used in Joints 1, 3A, 4 and 5. Joint No. 6 required a special lot due to its 8 in. grip. Tension shear jig tests were conducted to determine the shear strength and ultimate shear deformation.[11] The calibration test results are summarized in Table 2.

The bolts used in Joints 7 and 8 were from the same lot used in the Modified Bolted Joint.[9] Since the grip in the Modified Bolted Joint was 4-1/2 inches, Lot G bolts, originally made for this grip, slightly underfit the 5 inch grip required in Joints 7 and 8. There was insufficient bolt extension outside the plates to engage the full thread of the nuts as observed in the sawed section of Joint 7 shown in Fig. 9. A recess of about 3/16 in. occurred at the ends of the bolts. Both shear and tension calibration tests were conducted using the 5 inch grip to determine its effect upon the shear strength and clamping force. The results are summarized in Table 2 and are compared to the results for the normal 4-1/2 inch grip with a full nut.[6,9] The bolts provided the same shear strength for both grip lengths since no shear plane intersected the threads. However, a 10% decrease in torque tensile strength was observed. The average clamping force at 1/2 turn was similarly affected as shown in Table 2. The average clamping force of 45.5 kips measured in Joints 7 and 8 still exceeded the specified minimum tension in spite of the lack of full nut engagement. Shear and tension calibration tests for the Huck fasteners were also conducted. The results are listed in Table 3.

The instrumentation of the test specimens was similar to that reported in Ref. 9. All of the test joints were loaded to failure in static tension. A 5,000,000 lb. universal testing machine with flat wedge grips was used. The procedure used was similar to earlier studies.[1,6,9]

## TEST RESULTS AND DISCUSSION

Slip Behavior:- Figure 4 shows the load-deformation behavior of Joint 1 which was typical of the behavior observed in the other test joints. The shingle joints normally exhibited two separate load levels or stages at which major slip occurred. At the first slip load, substantial rigid body movement occurred along the shear plane adjacent to the main plate terminations with little or no movement along the second shear plane. The overall elongation at the first slip was about 50% of the total bolt-hole clearance. At the second slip load, rigid body movement was experienced along the second shear plane with some additional slip occurring along the first shear plane. The total overall movement was always less than the bolt-hole clearance.

Table 3-A compares the overall slip behavior of the test joints. Listed are the joint clamping forces and the first and second major slip loads. Values of the slip coefficient, $K_s$, corresponding to the first major slip load are given, assuming (a) two equal shear planes and (b) double shear in the first region and single shear in the interior regions (called effective slip in Col. 7).

It appears that assuming two equal shear planes when computing the slip coefficient for shingle joints can be misleading. Unlike butt joints, the transfer of load in shingle joints is not equal along each shear plane due to the unsymmetric positioning of plate terminations. This may lead to premature slip along one or more slip planes. (A relatively low slip coefficient could then be indicated for the joint by assuming an equal shear transfer). Slip coefficients below 0.3 were found for Joints 1, 4, 5, 7 and 8 assuming an equal shear transfer. The effective slip coefficients (Col. 7) computed on the assumption of double shear in the first region and single shear in the interior regions, are in better agreement with other test data.

It was evident that the shorter, stiffer shingle joints with high $A_n/A_s$ ratios, such as Joints 2, 3A and 3B, provided the greatest slip resistance. In the other joints, major slip first occurred at loads corresponding to somewhat lower slip coeffieients.

The least slip resistance was observed to occur in Joint 7 where the slip coefficient, assuming double shear, was 0.15. In the previous large bolted shingle joint which had comparable plate area but half the number of fasteners, a slip coefficient of 0.31 was reported.[6] This was nearly twice the value found for Joint 7 indicating that the slip resistance was not increased by doubling the number of fasteners.

In joint 7 slip only occurred along the shear plane adjacent to the plate terminations. No rigid body movement was observed along the other shear plane. The total amount of slip was small, amounting to about 50% of the total bolt-hole clearance. This was comparable to the amount of slip observed in the earlier large bolted shingle joint.[6]

Joint Strength:- The shingle joints tested in this series exhibited two distinct types of behavior in the non-linear range. Those with relatively high $A_n/A_s$ ratios provided load-deformation curves with relatively little non-linear deformation, as shown in Fig. 4. This behavior was also typical for Joints 2, 5 and 6. Multiple bolt shear failures occurred in these joints. As shown in Fig. 5, all six bolts in the first region of Joint 1 were sheared. In Joint 5 all the fasteners in the interior regions were sheared. Complete shear failures were observed in Joints 2, 3A and 3B. The ultimate loads for all test joints are listed in Table 3B.

The second type of observed behavior occurred in Joints 4, 7, and 8 which had lower $A_n/A_s$ ratios. The load-deformation curves were characterized by a long flat portion after gross section yielding. A typical load-deformation curve of this type is illustrated in Ref. 5. In these joints failure occurred by either a shearing off of the end fastener accompanied by necking in the main plates or by fracture of the plates.

Figure 6 shows the sawed section of Joint 8 illustrating the fastener deformation after failure. The end fastener had sheared off along the shear plane adjacent to the plate cut-offs. The amount of bolt deformation decreased rapidly from the end fastener toward the middle of the joint confirming that the end fasteners were critical. An apparent double shear condition existed in the first 6 or 7 fasteners of Region 1, as indicated by the deformation along both shear planes. Thereafter, the fasteners appeared to be essentially in single shear, transferring load primarily to the lap plates adjacent to the main plate cut-offs. Comparable behavior occurred in Joints 1, 2, 5 and 7, all having 3 regions.

In the 2-region joints (3A, 3B and 4), it was apparent that the load transfer continued along both shear planes in the interior region. The shear transfer along the bottom shear plane, however, was about 2/3 the amount transferred along the plane adjacent to the plate terminations.

Comparison of Theoretical Solution to Test Results:- The theoretical ultimate strength of the test joints was determined assuming, (a) complete double shear behavior, and (b) double shear behavior in Region 1 with single shear in the interior regions. In Table 3-B, the theoretical predictions are compared with the test results.

Except for Joint 2, the predicted ultimate loads assuming complete double shear were within 10% of the experimental values. The largest variations were in the shorter joints with high $A_n/A_s$ ratios. The strengths of these joints were overestimated.

In the 3-region joints, the tests showed (Fig. 6) that a condition close to double shear occurred in the first region but that single shear was more evident in the interior regions. The analytical predictions of the joint strengths assuming this type of behavior were comparable to the predictions assuming complete double shear in Joints 1, 5, 6, 7 and 8 (compare columns 4 and 5 in Table 4-B). In Joint 2 with a relatively high $A_n/A_s$ ratio, a substantial decrease in strength was predicted by assuming single shear in the interior regions. This was in agreement with the experimental results. Decreases in strength were also predicted in Joints 3A, 3B and 4. Since the actual behavior in the interior regions of these 2-region joints was closer to double shear, the theoretical predictions assuming single shear were conservative.

Columns 6 and 7 of Table 3-B compare the $A_n/A_s$ ratios of the test joints that correspond to the assumptions of complete and modified double shear. An increase in $A_n/A_s$ ratio resulted from the effective fastener shear area. The $A_n/A_s$ ratio in Joint 1 was 0.75 assuming double shear. This corresponds to an effective shear stress of about 22 ksi at the plate working load level. Single shear in the interior regions corresponds to an effective shear stress of 34 ksi. This is only slightly greater than the recommended value of 30 ksi for bearing type joints suggested in Ref. 4. The predicted strength of Joint 1 was not greatly influenced by the number of shear planes in the interior regions.

The effective shear stress in Joints 2, 3A and 3B corresponding to double shear in Region 1 with single shear in the interior regions is 45 ksi. Thus, the geometries of these joints are not likely to occur since excessively high allowable shear stresses would result. It is also unlikely for a joint with only 12 fasteners in line to be designed as a shingle joint.

Figure 4 compares the predicted load-deformation curve with the test results for Joint No. 1. The predicted joint deformation at various load levels was determined by integrating the computed bolt and plate deformations along the joint length. Where appropriate, the measured slip was added to the computed deformation. The theoretical curve followed the test results up to the predicted ultimate load of 1149 kips. The joint sustained further loading and continued to deform until failure occurred in the end fasteners at 1210 kips. The predicted ultimate strength was within 5% of the experimental value.

Comparison with Earlier Studies:- Table 4-B compares the predicted behavior of Joints 7 and 8 with a gage strip from the Modified Bolted Joint.[9] The Modified Bolted Joint had 16 fasteners in line with a distribution of 5, 5 and 6 fasteners per region. With comparable plate areas, Joint 7 had twice the total number of fasteners, and Joint 8 had twice the number in the first region.

The effective shear stress in Joint 7 at the plate working load level assuming double shear in Region 1 and single shear in the interior regions was about 14 ksi. This was comparable to the stress commonly used in bridge joints. In the Modified Bolted Joint the stress was analogous to that of a bearing type joint using the allowable stress recommended in Ref. 4.

The ultimate strengths were predicted assuming (a) complete double shear and (b) double shear in Region 1 with single shear in the interior regions. The predictions for the two types of behavior were comparable as shown in Columns 4 and 5 of Table4-B. An increase in strength of about 12% was predicted by doubling the number of fasteners in the Modified Bolted Joint. The same effect was predicted by doubling the number of fasteners in only the first region. The experimental results were in good agreement with the analytical predictions. As reported in Ref. 3, the predicted strength of the complete Modified Bolted Joint assuming double shear was within 5% of the experiment load of 3550 kips.

It was concluded that the strength of large bridge joints would not be significantly decreased by removing up to half the number of fasteners currently used in design.

Comparison of A325 and Huck Fastened Joints:- The slip behavior of the Huck-bolted joints was comparable to the behavior observed in the A325 bolted joints. Joints 3A and 3B were identical except for the type of fastener. The difference in slip loads as shown in Table 3-A was about 12%. Slightly lower clamping forces were found in the Huck fasteners than in the A325 bolts tightened by the turn-of-nut method as shown in Table 2. The clamping forces induced in the Huckbolt fasteners, however, were in excess of the minimum requirements of A325 bolts.

The shear jig tests results indicated that the double shear strength and ultimate deformation were nearly identical for the A325 bolts and Huckbolts. Joints 3A and 3B had the same geometry and their predicted and measured ultimate loads were nearly identical as shown in Table 3-B.

## DESIGN OF SHINGLE JOINTS

Approximate Methods of Analysis:- Shingle joints like other types of connections are statically indeterminant. The condition is further complicated in shingle joints by the unsymmetric positioning of main plate terminations. Analytical elastic solutions that predict the distribution of load in the main and splice plates of shingle joints have been developed.[9] The solution has been extended into the plastic range so as to predict the ultimate strength of the connection.[3] These theoretical analyses are too cumbersome and impractical for ordinary design practice.

There are several existing methods for estimating the distribution of force in the main and lap plates of a shingle splice. Two of the most popular methods are:[12]

1. Forces in splice plates are inversely proportional to their distances from the member being spliced.
2. Forces in each member at a section through a splice are proportional to their areas.

In Method 1, it is assumed at each discontinuity that the amount of force distributed to the lap plates is proportional to the area of the member being terminated. The forces in the continuous main members are assumed to remain unchanged. The transfer of load is made in the region directly preceding the point of termination and it is assumed that the original load is restored to the spliced member in the region following the termination.

In Method 2, the total applied load is assumed to be distributed to all continuous members at the position of a main plate termination in proportion to their areas. No direct assumption is made regarding the amount of load transferred to the splice plates in a particular region as in Method 1. If the lap plates are of equal area, Method 2 predicts that the shear transfer is equal along the top and bottom shear planes in the first region regardless of their positions with respect to the member being terminated.

The test results reported herein and previous shingle joint tests have shown that at each plate discontinuity, there was a sudden pick-up of load in the adjacent plate elements.[6,9] A third method of analysis was developed on the basis of these observations. This method assumes that the total load is distributed to all members at a section through the joint in proportion to their areas, first considering the terminated members as being continuous. The load assumed to be carried by a terminating member is then distributed to the two adjacent plates in proportion to their areas.

<u>Comparison of Design Methods with Test Results</u>:- The partition of load in the test joints was determined from the measured plate strains at different cross sections along the joint lengths.

Figure 7 compares the measured plate forces in Joint 1 with the various design methods. Similar distributions were found in the other test joints. The comparison was made at the working load level of the main plate.

The top portion of the figure compares the design curves with the measured forces in the combined top lap plates. The central portion makes a similar comparison for the main plate component, and the lower portion compares the results for the lower lap plates.

In Fig. 8, the various methods for design are compared with the test results reported in Ref. 9 for the Modified Bolted Joint. The comparisons are made for the top lap plate, main plate and bottom lap plate components at the 2080 kip working load level.

The geometry of the Modified Bolted Joint differed from the geometries of the test joints in that it had two splice plates along the bottom shear plane. Only a single bottom splice plate was used with the test joints reported herein.

It is apparent from the test results that Method 1 substantially underestimated the total transfer of load in the first region. The measured load transferred to the splice plates always exceeded the proportion of main plate area initially being terminated. In Joint 1 (Fig. 7), 50% of the applied load was transferred to the lap plates in the first region although only 33% of the main plate was terminated. In the Modified Bolted

Joint, over 50% of the applied load was initially distributed.

Loads substantially greater than estimated by Method 1 were measured in the bottom lap plates of all joints. The test results indicated that the forces in the top and bottom lap plates were nearly equal in the first region. In the Modified Bolted Joint more load was actually measured in the combined bottom lap plates than in the top lap plate as shown in Fig. 8.

The greatest variation between the load partition determined by Method 1 and the test results occurred in the Modified Bolted Joint (Fig. 8). It was apparent that the assumptions in Method 1 used to determine the distribution of force to the lap plates were not very satisfactory.

The distributions of load in the main plates predicted by Method 2 were in good overall agreement with the measured forces. In Joint 1, it was estimated that 50% of the load would be distributed to the lap plates in the first region. In the Modified Bolted Joint (Fig. 8), because of the greater proportion of splice material, it was estimated that 59% of the load would be distributed. Both assumed distributions were comparable to the test results.

Slight variation between the distributions determined by Method 2 and the test results occurred in the top and bottom lap plates. The forces in plates adjacent to a plate termination were slightly underestimated in all test joints. The greatest deviations were observed in the top lap plates adjacent to the first plate terminations and in the bottom lap plates adjacent to the final plate terminations. Increases in plate loads occurred at those points.

The distributions of force determined by Method 3 provided the best correlation with the test results, as shown in Figs. 7 and 8. The method provided a reasonable estimate of the force distributions in all joint compenents. This method accurately predicts a more effective use of the fasteners in the interior regions and thus requires less fasteners than the other methods.

It is recommended for design, that Method 3 be used to approximate the load distribution in the plates and fasteners. With this method, it is also recommended that the first region of shingle splices have double lap plates of equal area. This reduces the critical shear transfer along the plane adjacent to the first plate termination.

Where practical, it is also recommended that the top and bottom lap plates have equal lengths in the first region. As shown in Fig. 6, equal deformation was observed along both shear planes at failure. It is believed that equal length splice plates would more effectively utilize the critical end fasteners.

With the introduction of a gusset into the splice as in a truss joint, however, additional fasteners are required along the shear plane adjacent to the gusset to transfer load from diagonal members. Since these fasteners are not required along the bottom shear plane, it is believed that the bottom lap plates can be shorter than the top lap plate in the first region if an adequate number of fasteners is still provided. Since Method 3 more accurately predicts the shear transfer along the bottom plane, it is recommended for use.

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## SUMMARY AND CONCLUSIONS

These conclusions are based on analytical studies and a complimentary experimental program that examined the behavior and ultimate strength of shingle joints.

(1) The analytical studies showed that A325 bolts were capable of satisfactory behavior ~~at allo~~wable shear stresses up to 40% higher than used in current practice.

(2) At high $A_n/A_s$ ratios, one-region butt joints were predicted to be more efficient than shingle joints with two and three regions due to the complete double-shear behavior and constant joint stiffness in the butt joints. At lower $A_n/A_s$ ratios, the number of regions had no effect upon the predicted strength.

(3) The test results confirmed the indications of previous studies that slip in shingle joints tends to be less than the hole clearance. Since shingle joints are most often used where reversal of stress is unlikely because of large dead loads, it appears reasonable to assume that shingle joints are not slip-critical.

(4) Two distinct types of behavior in the non-linear range were encountered. Joints with high $A_n/A_s$ ratios

experienced relatively little non-linear deformation and resulted in multiple-bolt shear failures. In joints with lower $A_n/A_s$ ratios, large deformations were measured after gross section yielding. End-bolt shear failures or plate failures were observed.

(5) In the 3-region joints, the tests showed that a condition close to double shear occurred in the first region but that single shear was more evident in the interior regions. In the 2-region joints, it was apparent that the load transfer continued along both shear planes in the interior regions.

(6) The load distributed to the splice plates at the position of a main plate termination was more proportional to the area of the splice plates at that point than to the area of the main plate being terminated. A sudden pick-up in load was measured in the plates directly adjacent to a plate discontinuity.

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(7) The distribution of load determined by Method 1 can be misleading for design, since the method substantially underestimates the load carried by the splice plates furthest from the plate discontinuities.

(8) A reasonable approximation of the load partition in shingle joints can be made by assuming the force in all continuous members at the position of a main plate termination to be proportional to their areas.

(9) A more exact approximation can be made using the two-stage distribution, described in the text as "Method 3," taking into account the sudden pick-up of load in plates directly adjacent to a plate termination.

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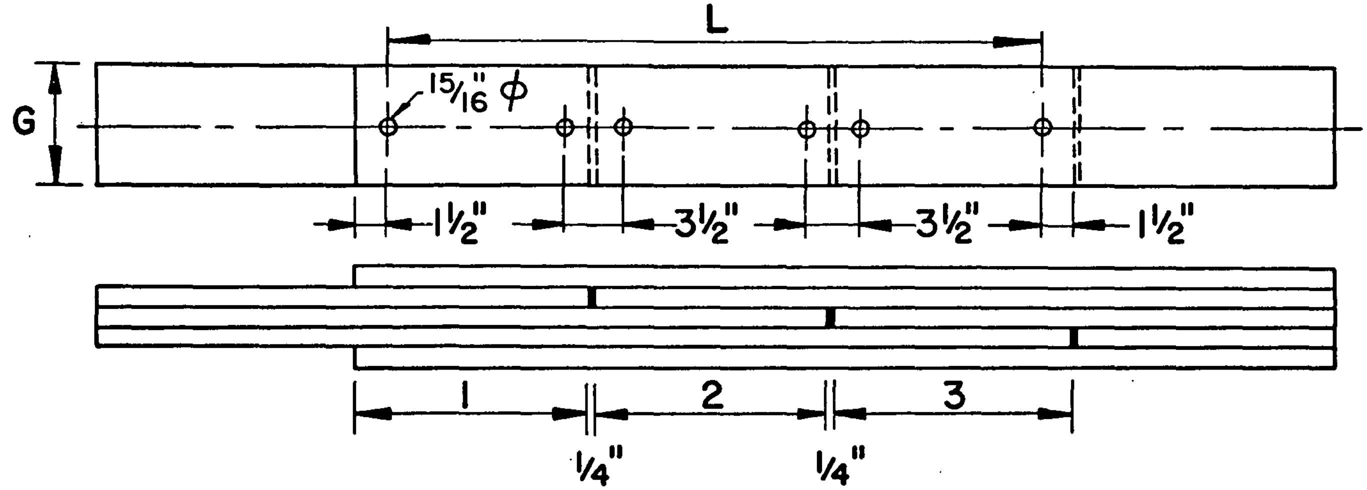
## TABLE I-A DESCRIPTION OF TEST JOINTS

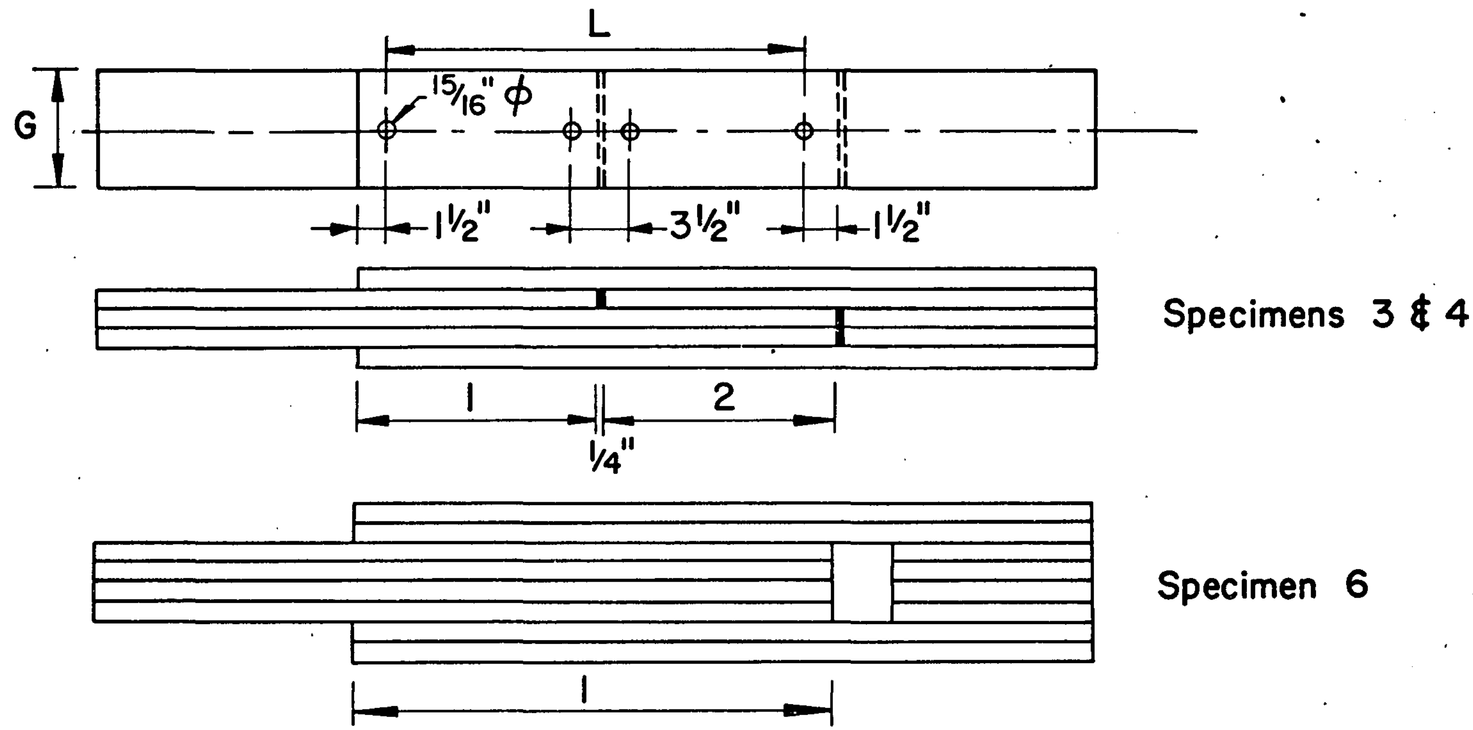



Specimens 1, 2, 5, 7, & 8

| TEST JOINT | NUMBER OF FASTENERS | FASTENERS PER REGION | | | LENGTH L (INCHES) | REGION LENGTHS (INCHES) | | | GAGE G (INCHES) |
|---|---|---|---|---|---|---|---|---|---|
| | | 1 | 2 | 3 | | 1 | 2 | 3 | |
| 1 | 18 | 6 | 6 | 6 | 52.0 | 18.125 | 18.250 | 18.125 | 6.375 |
| *2 | 12 | 6 | 3 | 3 | 34.0 | 18.125 | 9.250 | 9.125 | 6.375 |
| 5 | 18 | 10 | 4 | 4 | 52.0 | 30.125 | 12.250 | 12.125 | 6.375 |
| 7 | 32 | 10 | 10 | 12 | 94.0 | 30.125 | 30.250 | 36.125 | 5.000 |
| 8 | 21 | 10 | 5 | 6 | 61.0 | 30.125 | 15.250 | 18.125 | 5.000 |

* Joints using Huckbolt fasteners

# TABLE I-B DESCRIPTION OF TEST JOINTS




| TEST JOINT | NUMBER OF FASTENERS | FASTENERS PER REGION | | LENGTH L (INCHES) | REGION LENGTHS (INCHES) | | GAGE G (INCHES) |
|---|---|---|---|---|---|---|---|
| | | 1 | 2 | | 1 | 2 | |
| 3-A<br>*3-B | 12 | 6 | 6 | 33.5 | 18.125 | 18.125 | 6.375 |
| 4 | 12 | 6 | 6 | 33.5 | 18.125 | 18.125 | 4.000 |
| 6 | 12 | 12 | - | 33.0 | 36.000 | - | 5.000 |

* Joints using Huckbolt fasteners

TABLE 2: FASTENER MATERIAL PROPERTIES

| Bolt Lot (1) | Used in Joint (2) | Shear Jig: Ultimate Double-Shear Kips (3) | Shear Jig: Ultimate Shear Stress ksi (4) | Shear Jig: Ultimate Deformation Inches (5) | Direct Tension Tensile Strength ksi (6) | Torque Tension Tensile Strength ksi (7) | Average Clamping Force Per Bolt Kips (8) |
|---|---|---|---|---|---|---|---|
| XA | 1,3A,4,5 | 106.0 | 88.1 | 0.176 | 146.2 | 130.0 | 58.9 |
| SA | 6 | 107.8 | 89.5 | 0.181 | 144.0 | 116.9 | 53.9 |
| G** | M.B.J. | 91.5 | 76.0 | 0.148 | 127.7 | 111.5 | 51.0 |
| G | 7,8 | 94.1 | 78.2 | 0.205 | -- | 99.6 | 45.5 |
| Huck | 2,3B | 110.0 | 91.5 | 0.166 | -- | -- | 45.6* |

*Determined from installation in Skidmore-Whilhelm.

**Normal thread engagement, 4½ in. grip.

TABLE 3-A: TEST RESULTS - SLIP BEHAVIOR

| Test Joint (1) | Number of Fasteners (2) | Clamping Force Kips (3) | 1st Slip Load Kips (4) | 2nd Slip Load Kips (5) | $K_s$ Double Shear (6) | $K_s$ Effective (7) |
|---|---|---|---|---|---|---|
| 1 | 18 | 1060 | 504 | 590 | 0.24 | 0.36 |
| 2* | 12 | 547** | 360 | 470 | 0.33 | 0.44 |
| 3A | 12 | 703 | 508 | 600 | 0.36 | 0.48 |
| 3B | 12 | 547** | 446 | 504 | 0.41 | 0.54 |
| 4 | 12 | 713 | 340 | -- | 0.24 | 0.32 |
| 5 | 12 | 1055 | 560 | 1040 | 0.26 | 0.35 |
| 6 | 12 | 646 | 366 | -- | 0.28 | 0.28 |
| 7 | 32 | 1455 | 416 | -- | 0.15 | 0.22 |
| 8 | 21 | 955 | 438 | 574 | 0.23 | 0.31 |

*Joints fastened with Huckbolt Fasteners.

**Determined from mean clamping force of individual fasteners in Skidmore-Whilhelm. Actual clamping forces may be 10-15% higher.

TABLE 3-B: TEST RESULTS - ULTIMATE STRENGTH

| Test Joint | Failure Mode | Recorded Ultimate Kips | Predicted Ultimate | | $A_n/A_s$ | | Effective Shear Stresses | |
|---|---|---|---|---|---|---|---|---|
| | | | Double Shear Kips | Modified Shear Kips | Double Shear | Modified Shear | Double Shear ksi | Modified Shear ksi |
| (1) | (2) | (3) | (4) | (5) | (6) | (7) | (8) | (9) |
| 1 | Bolts | 1210 | 1150 | 1124 | .75 | 1.13 | 22.5 | 34 |
| 2 | Bolts | 982 | 1129 | 943 | 1.13 | 1.52 | 34 | 45 |
| 3A | Bolts | 1030 | 1128 | 904 | 1.13 | 1.52 | 34 | 45 |
| 3B | Bolts | 1044 | 1133 | 918 | 1.13 | 1.52 | 34 | 45 |
| 4 | Plates | 754 | 726 | 634 | .64 | .85 | 19.5 | 25 |
| 5 | Bolts | 1142 | 1179 | 1105 | .75 | .97 | 22.5 | 29 |
| 6 | Bolts | 1150 | 1128 | 1128 | 1.13 | 1.13 | 34 | 34 |
| 7 | Bolts Plates | 962 | 937 | 936 | .32 | .48 | 10 | 14 |
| 8 | Bolts | 932 | 937 | 936 | .48 | .65 | 14 | 20 |
| M.B.J. | Bolts | 888* | 841 | 832 | .64 | .98 | 19 | 29 |

*Taken as 25% of the test load (1 gage strip).

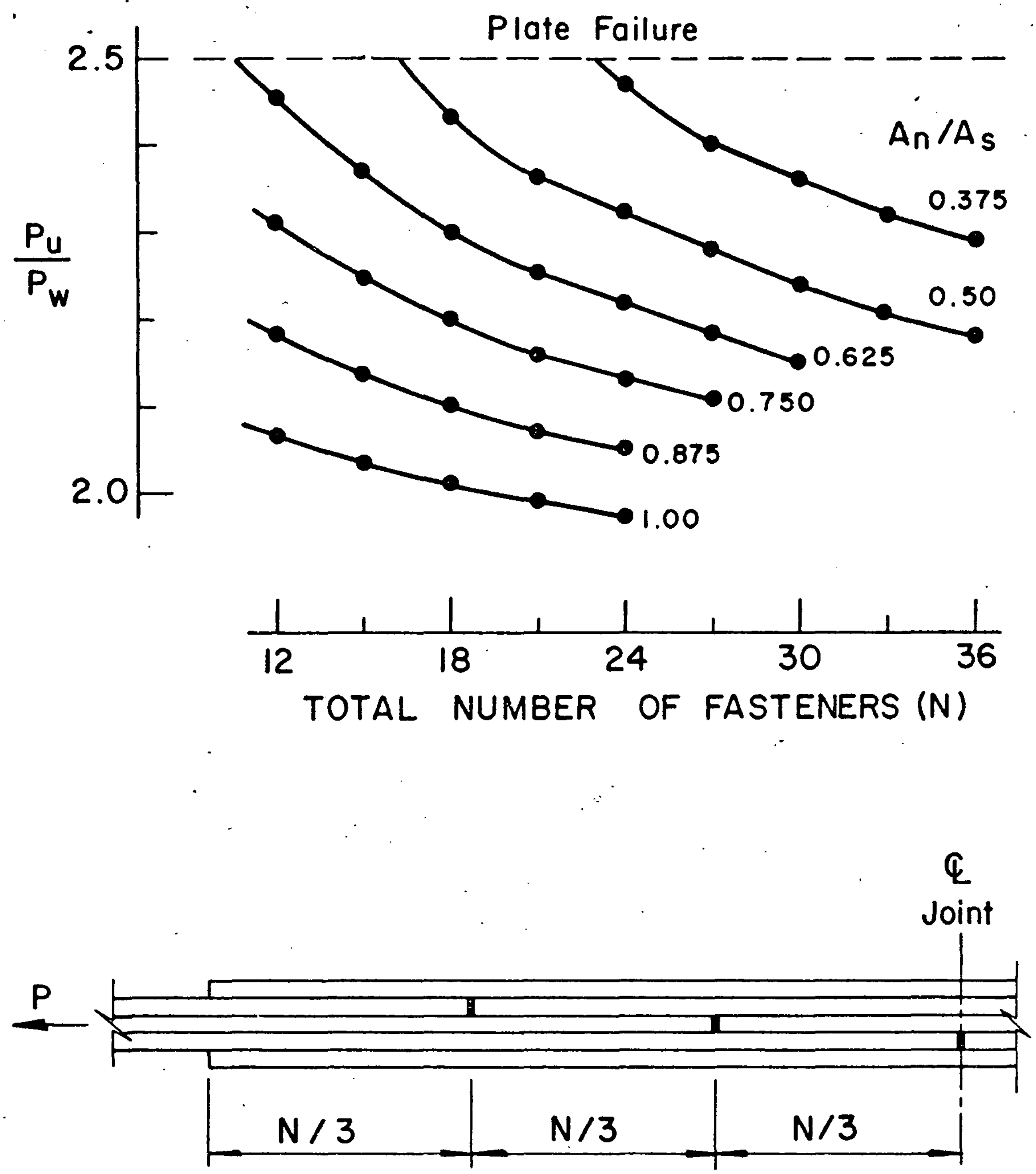



Fig. 1. Effect of Variation in $A_n/A_s$ Ratio and Joint Length Assuming Double Shear

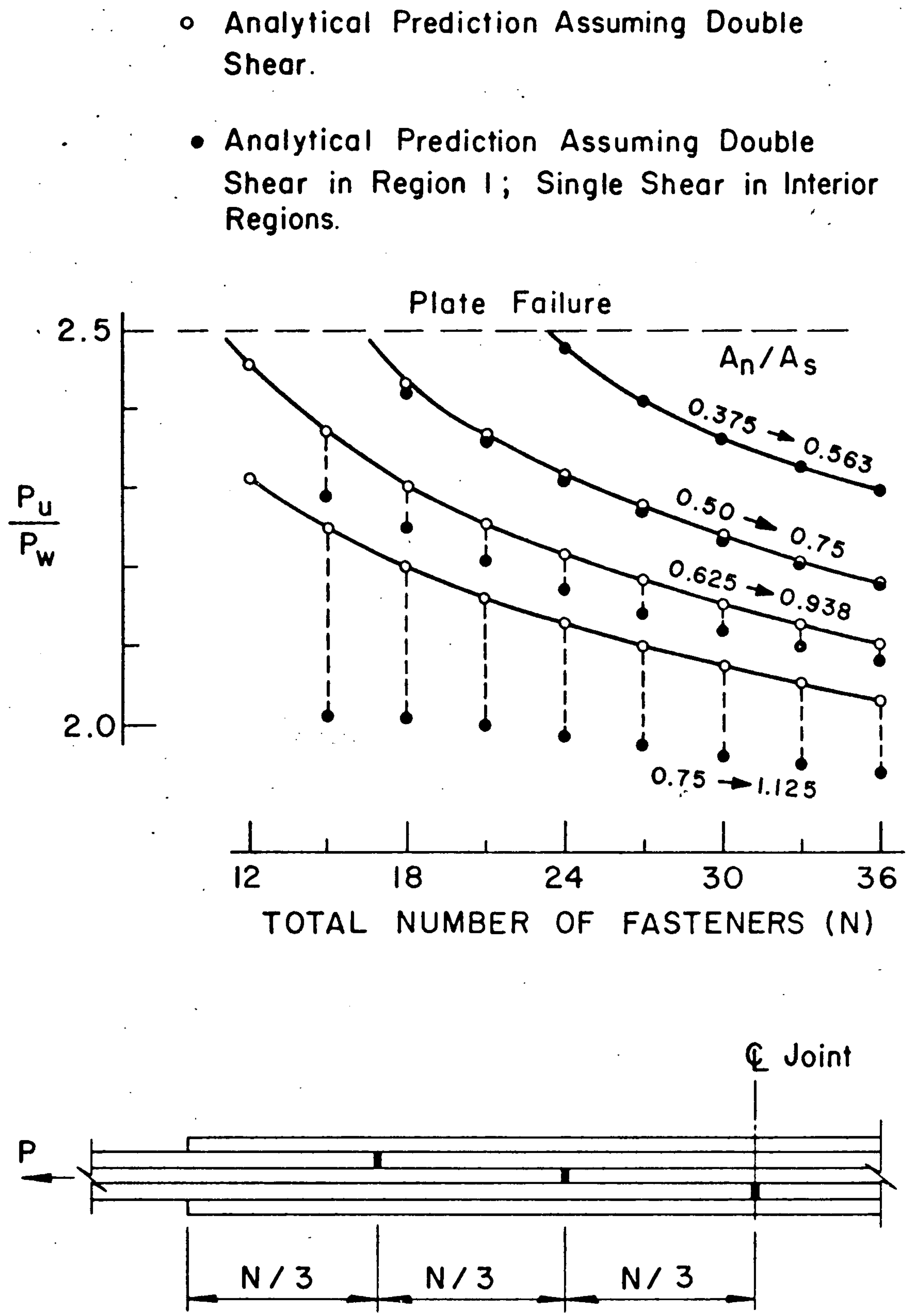



Fig. 2 Effect of Assuming Single Shear in Interior Regions

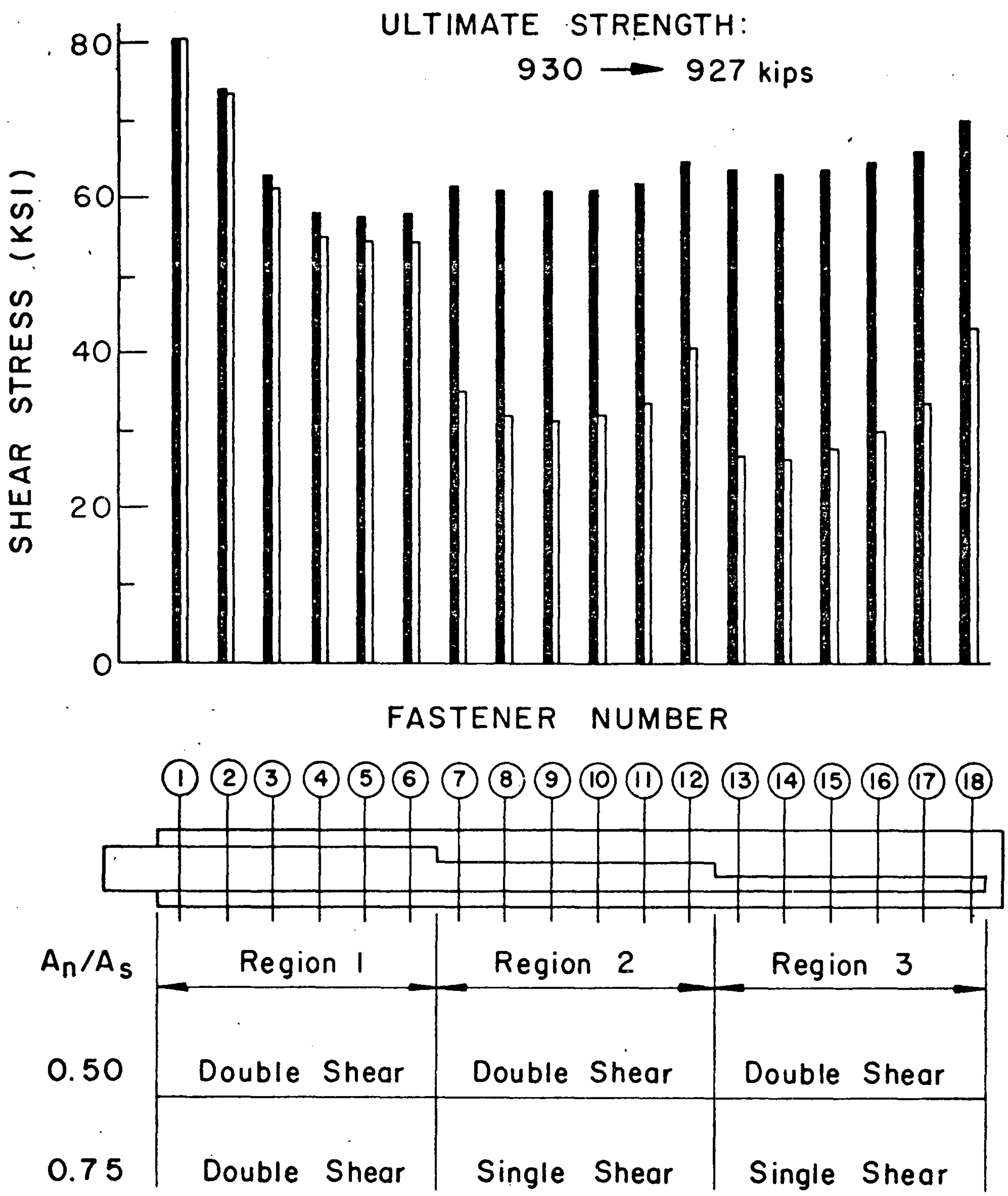



Fig. 3 Theoretical Fastener Stress Distribution

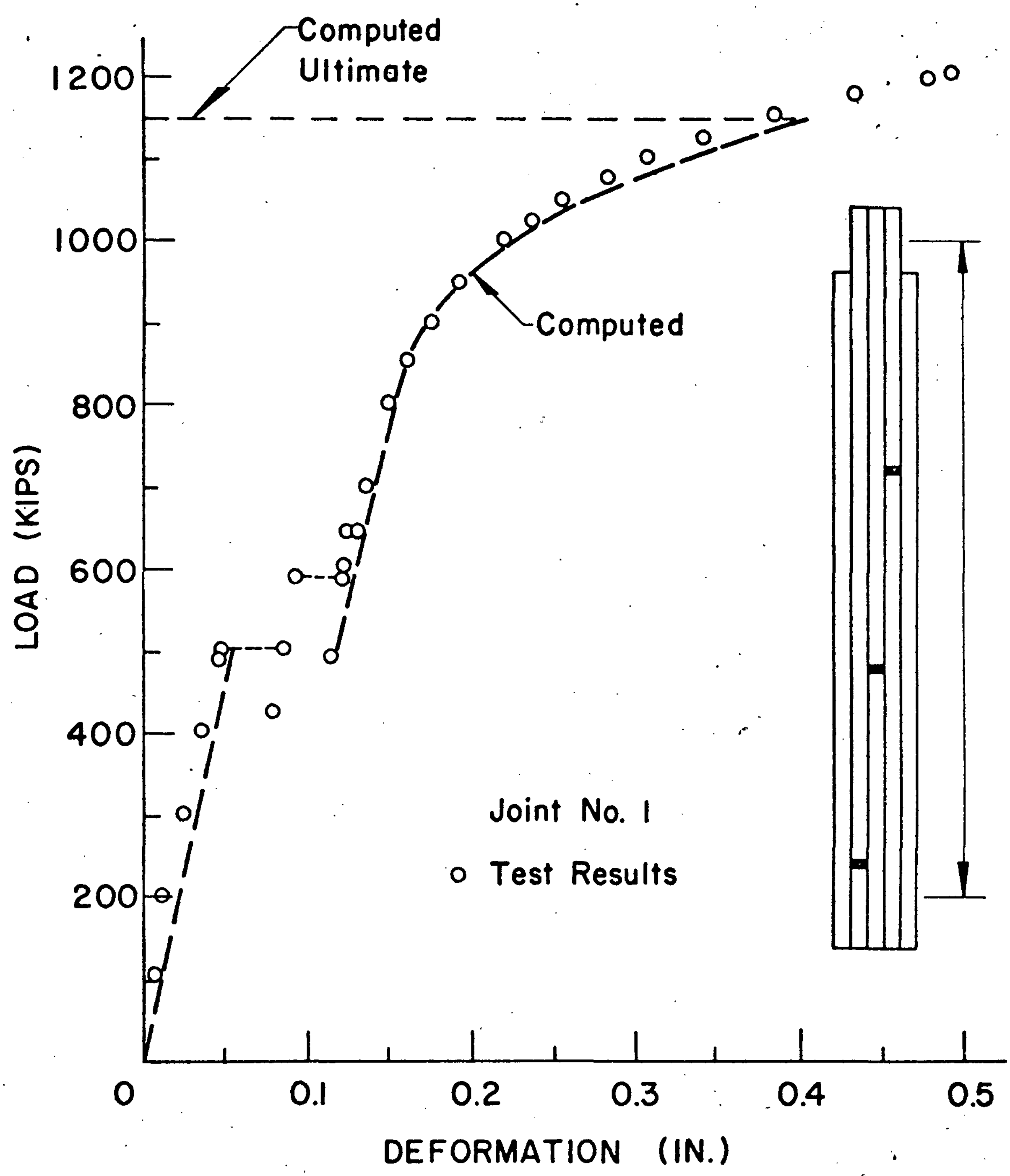
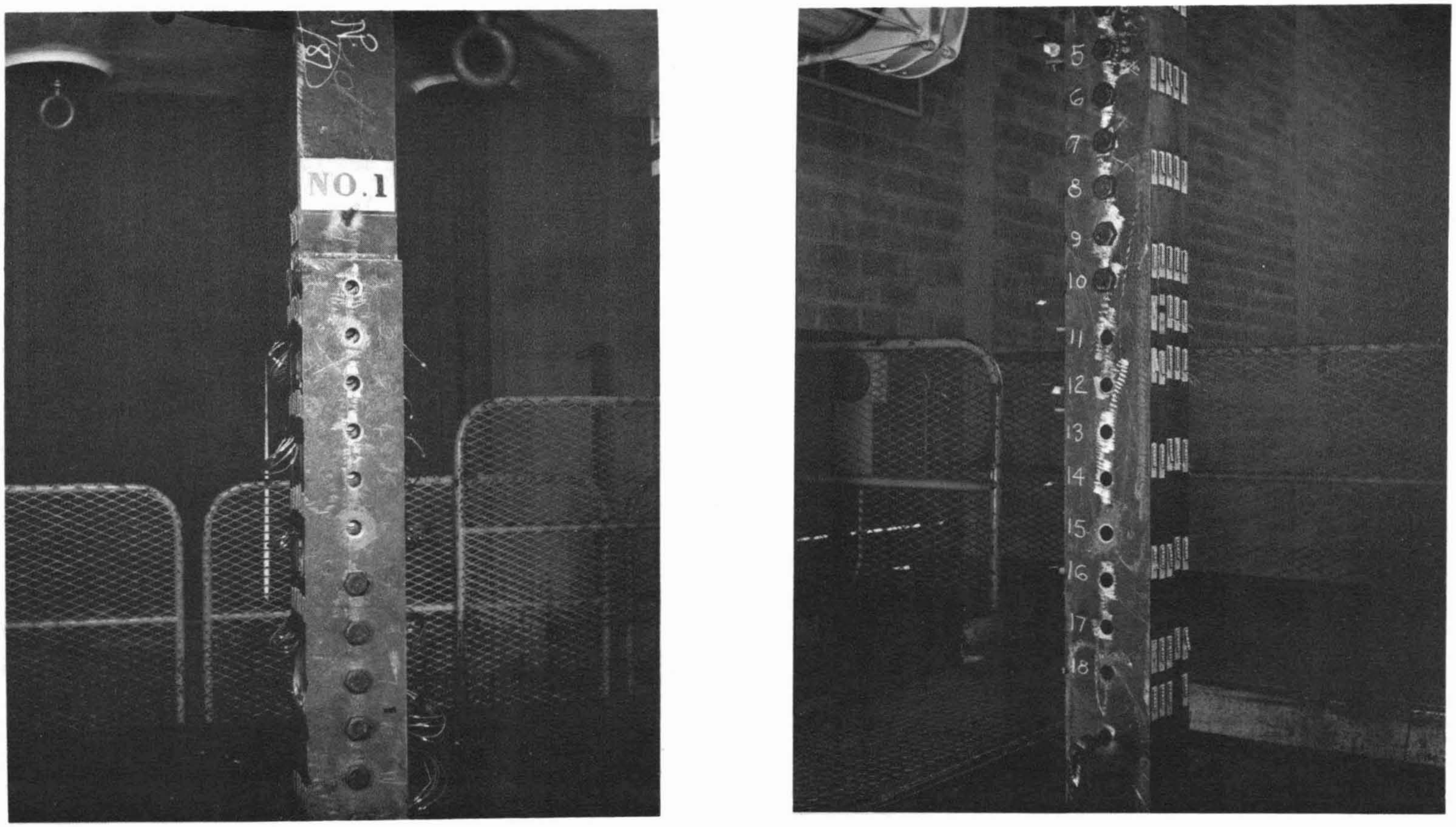



Fig. 4 Computed and Experimental Load-Deformation Behavior




(a) Failure in Region 1 of Joint 1

(b) Failure in Interior Regions of Joint 5

Fig. 5 Multiple Bolt Shear Failures

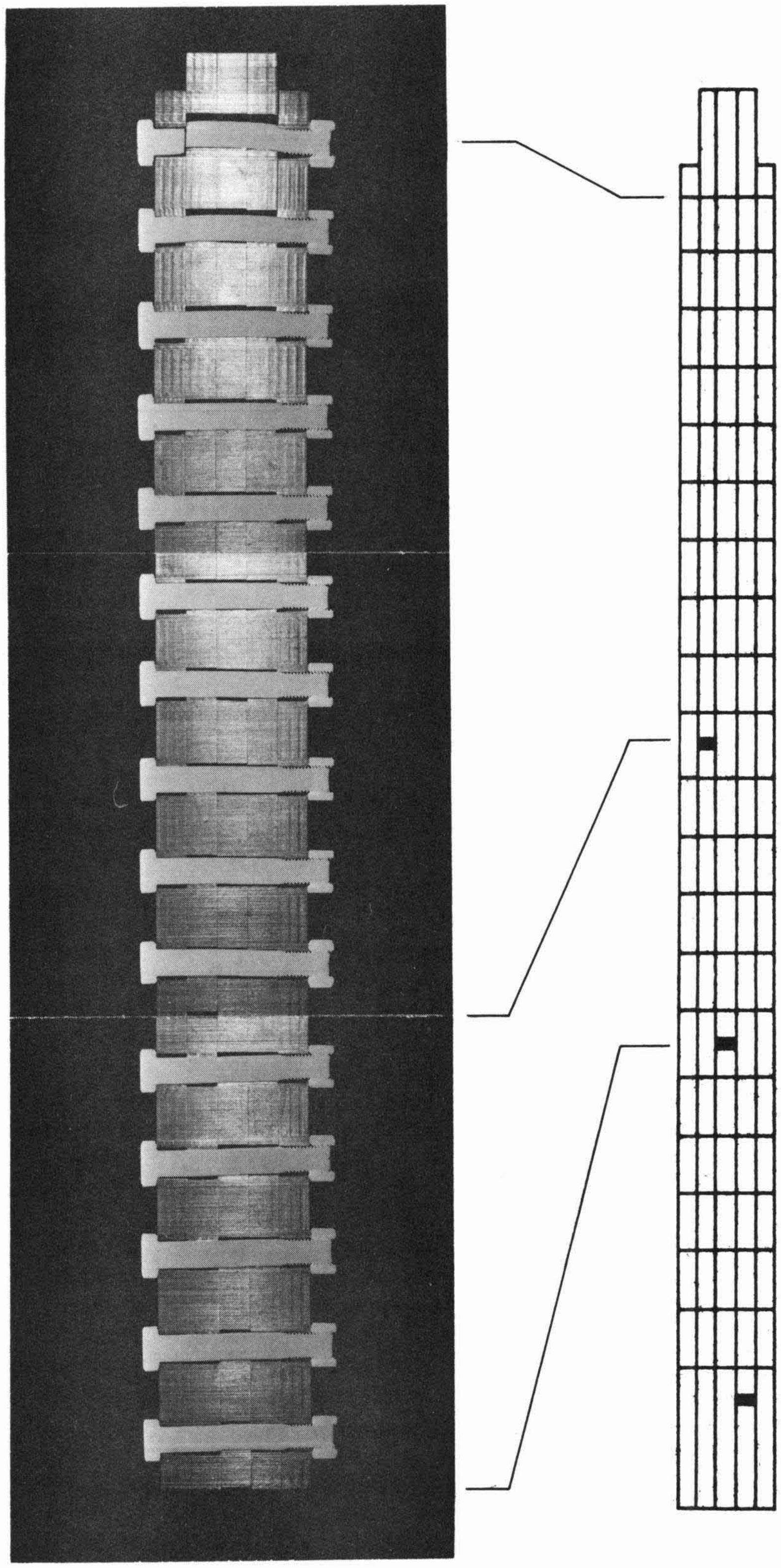


Fig. 6 Sawed Section of Joint 8 After Failure

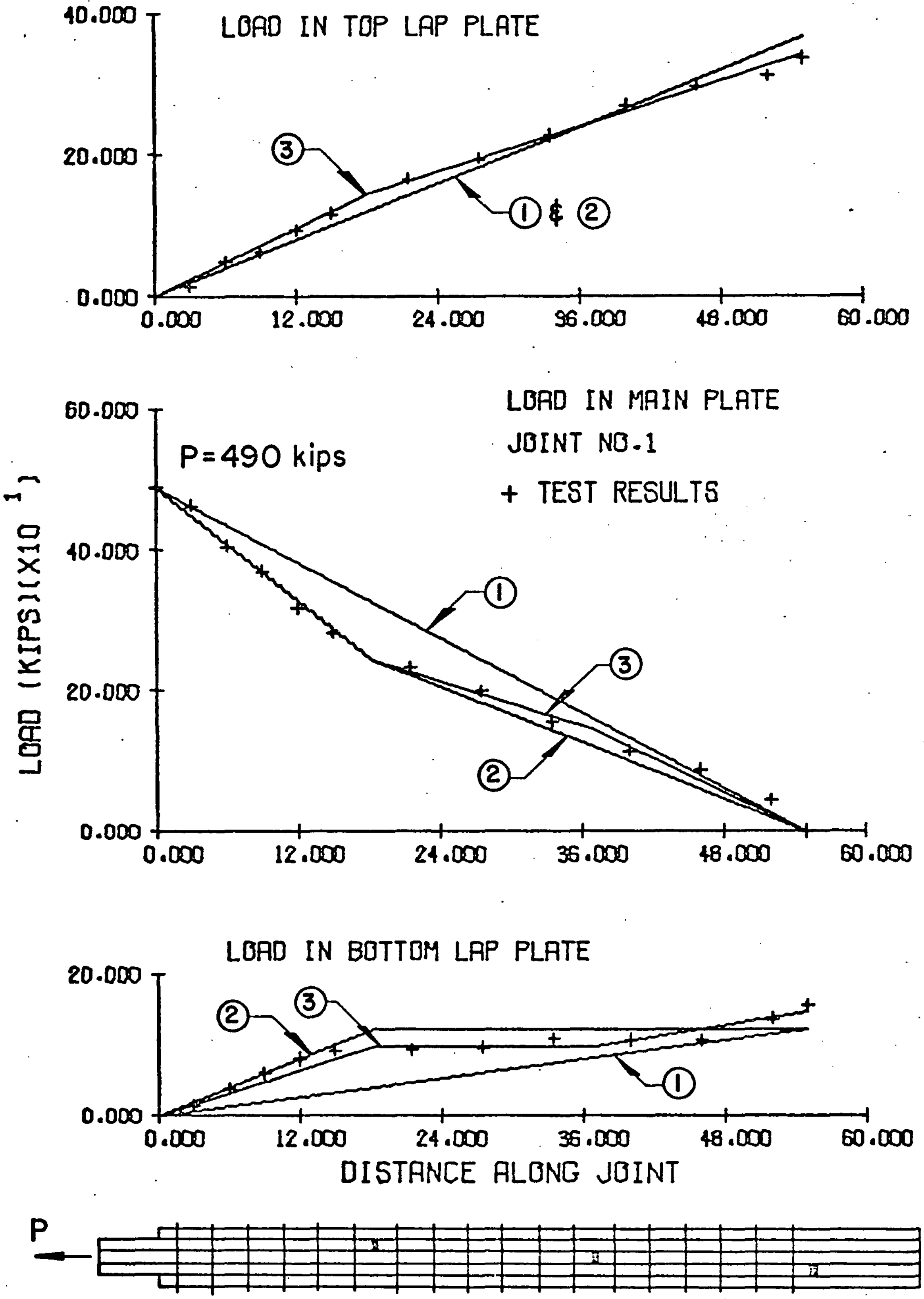



Fig. 7 Comparison of Design Methods with Test Results of Joint 1

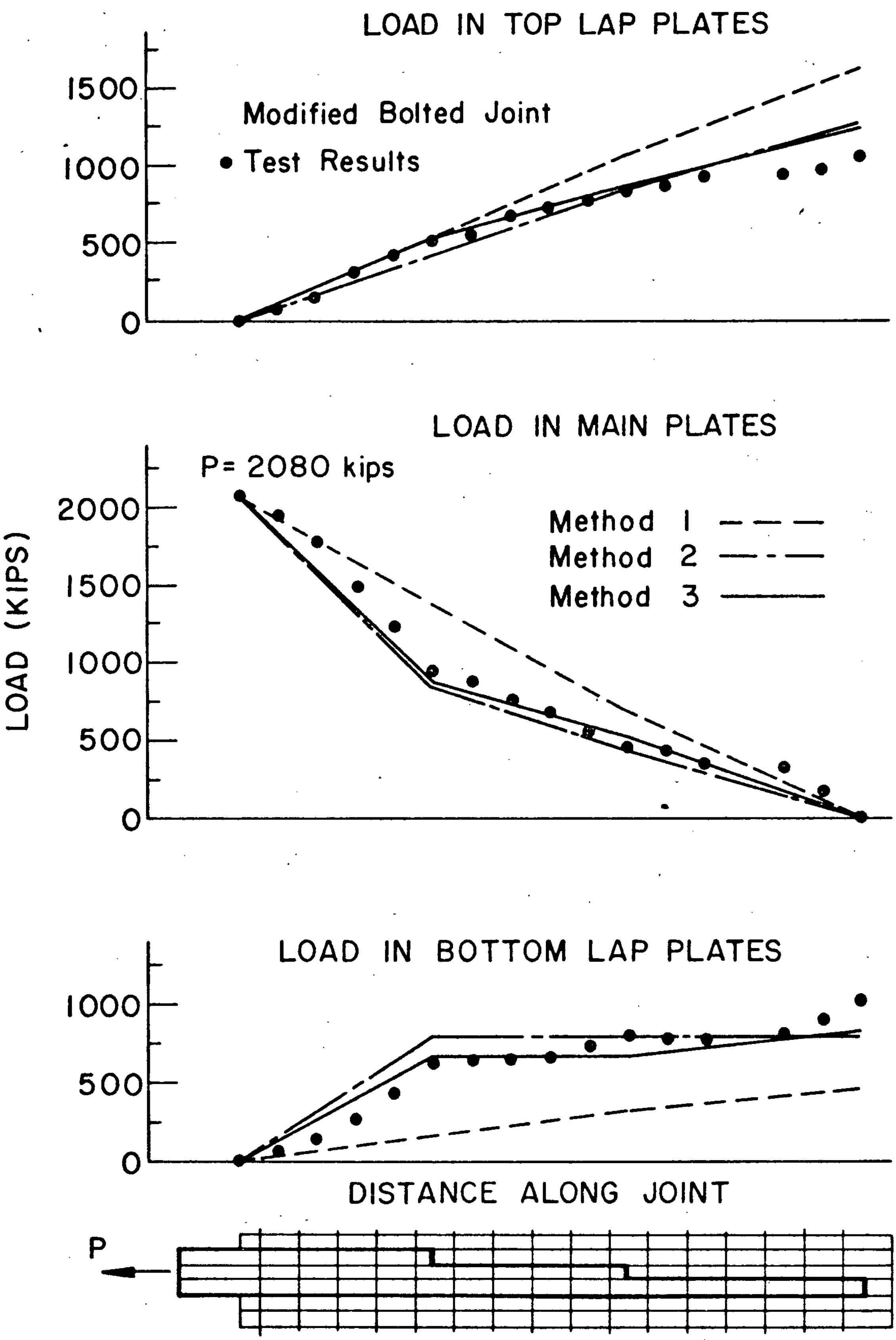



Fig. 8 Comparison of Design Methods with the Experimental Load Partition in the Modified Bolted Joint